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AMERICAN SOCIETY OF CIVIL ENGINEERS

OCTOBER, 1955

DISCUSSION OF PROCEEDINGS PAPERS

562, 649, 651, 652, 659

STRUCTURAL DIVISION

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Discussion of
"MOMENTS AND DEFLECTIONS IN STEEL GRID BRIDGE FLOORS"

by H. J. Greenberg
(Proc. Paper 562)

H. J. GREENBERG.[1]—Mr. Chu's approximate formulas have the advantage of being close in form to the AASHO formulas for the isotropic case besides being somewhat simpler for computation. The values furnished are in sufficiently good agreement with the tables given by Kuo in the paper cited previously. In reference to the limitations listed by the writer, the following remarks should be added. The results of Kuo's analysis were originally presented in terms of the non-dimensional parameter k which is the ratio of a side of a central square load to the span S (Fig. 2, 3). If the loaded area is assumed to be 1" on a side rather than 15" the corresponding formulas are obtained by changing S in the same proportion, i.e. $\frac{1}{15}$ S. Moreover, the exact assumption as to the shape of the loaded area, i.e. square, circular or other, does not significantly affect results except in extreme cases. Also since the exact shape of the loaded area is unknown, there is little reason to choose one shape over another except for purposes of mathematical simplicity. In connection with (2) and (5), for the case of equal panels the maximum moments and deflections will occur in the case of two panels loaded as shown in Fig. 2 or in the case of edge moments as in Fig. 4. The case of more than one load for longer spans could be handled in special cases by superposition or in any case by going back to Kuo's general treatment, which applies to an arbitrary panel in a one-way continuous, anisotropic slab under arbitrary loading, and carrying out the numerical computations from the series solution.

As Mr. Chang remarks, the formulas are on the conservative side for interior spans. Additional computations based on Kuo's theoretical solution would furnish the desired information here. The numerical difference solution suggested by Chang is always possible but as usual the accuracy cannot be determined. It would seem preferable in these problems to proceed from an exact solution as based on Kuo's results even though the solution is not immediately in a useful numerical form.

In a letter, Mr. T. A. Kae has pointed out that the formula (24) given by Kuo for the moment coefficient M_{cy}/P for the case of open grid floors is not in good agreement with the tabulated values in his paper. This may be due to a typographical error in the formula, but it has not been possible to check on this. The following approximate formula, however, has been found to furnish good agreement with the tabulated values

1. Associate Prof. of Math., Carnegie Inst. of Technology, Pittsburgh, Pa.

$$\frac{M_{cy}}{P} = (.0088\ S + .17)r^{\left(\frac{1.35}{S} + .24\right)}$$

for the range $.01 \leq r \leq .04$, $2.5' \leq S \leq 12.5'$.

Finally, the author would like to make acknowledgement to the Reliance Steel Products Company of McKeesport, Pa. with which the author is associated as research consultant and which sponsored these investigations.

Discussion of
"THE CONSTANT SEGMENT METHOD FOR THE ANALYSIS OF NON-UNIFORM STRUCTURAL MEMBERS"

by Walter E. Hanson and Wallace F. Wiley
(Proc. Paper 649)

T. F. HICKERSON,[1] M. ASCE.—Although not so stated by the authors, this paper is based on the Conjugate Beam method. Believing that this principle offers the best approach to structural analysis, particularly to beam slopes and deflections, the writer has used it exclusively in a similar work.[2]

Fig. 13

Comparing (a) and (b) of Fig. 13, it is obvious that $\theta_B^A = \theta_A^B$ without mentioning Maxwell's law of reciprocal relations.

At the bottom of Tables I through VI, the totals could have been given to advantage. Thus in Table I, from left to right Σ = .33333, .33333, .16667, indicating that as a limiting condition when I is constant throughout the member,

$$\theta_A^A = \frac{1}{3}\frac{L}{EI} \quad \text{and} \quad \theta_A^B = \frac{1}{6}\frac{L}{EI} \quad \text{for unit moment at A.}$$

Also, in Table II, for b = 0.4, Σ = .064. Hence if I is constant,

$$\theta_A^P = 0.064 \times \frac{PL^2}{EI}.$$

The stiffness, carry-over, and distribution factors are based on the assumption that the other end of the member is fixed—a condition that usually does not exist. Hence distribution and carry-over must take place through a few cycles in order to obtain the final correct result.

The inclusion of coefficients for deflections produced by a concentrated load would have added to the completeness of the paper. Perhaps the authors wanted to avoid dealing with absolute maximum deflections. However the mid-span deflection coefficients are closely representative of the maximum deflections.

1. Prof. of Applied Math., Univ. of North Carolina, Chapel Hill, N. C.
2. "Statically Indeterminate Frameworks," "Beam Deflection when I is Constant or Variable," University Press, Chapel Hill, N. C.

The neatness, clarity, and accuracy of the paper is certainly splendid. But doubt exists as to whether it presents the simplest and shortest procedure for obtaining slopes, deflections, or moments when I is variable.

EUGENE J. VAYDA,[3] M. ASCE.—The authors are to be commended for their interpretation and presentation of the subject.

The fixed end moment at the top of a haunched column that is bent by an offset P force, and hinged at the bottom, as shown in Fig. 1a shall be determined.

The elastic line of the simply supported beam AB bent by a moment M, where M = Pe, will have a slope of Θ_p with the horizontal at point B (Fig. 1b). The moment of inertia, I, shall be computed at the center of gravity of each division (Fig. 1c). After finding M_p and $\frac{M_p}{I}$ at the center of gravity of each division the reaction of the conjugate beam, loaded with the $\frac{M_p}{I}$ diagram, shall be R_p at point B (Fig. 1d). Then $E\Theta_p = R_p$, where E is the modulus of elasticity.

Let Θ_M be the slope of the elastic line at point B with the horizontal when the same simply supported beam is bent by a moment of $M_M = 1$ (Fig. 1e). After finding M_M and $\frac{M_M}{I}$ at the center of gravity of each division the reaction of the conjugate beam at point B, loaded with the $\frac{M_M}{I}$ diagram shall be R_M (Fig. 1f). Then $E\Theta_M = R_M$, and if the total slope of the elastic line at the fixed end B is Θ_B,

$$E\Theta_B = E(\Theta_p + \Theta_M) = R_p + R_M M_M = 0$$

and from this

$$M_M = -\frac{R_p}{R_M}$$

The graphical analysis presents here a fast solution as shown in the following example.

Let us assume that a crane runway attached to the vertical member of a rigid frame creates a bending moment of 56 ft. kips as shown in Fig. 2a. Column is hinged at the 1' - 8" wide end and fixed at the 2' - 4" wide end.

Values shown in Table 1 were computed on a 10" slide rule.

Determine M_p as shown in Fig. 2b for each division point. With the $\frac{M_p}{I}$ force polygon (Fig. 2f) draw the funicular polygon of Fig. 2c. The line drawn in Fig. 2f parallel to the X line of Fig. 2c will cut off the reaction of the conjugate beam. (R_p)

With the force polygon (Fig. 2e) draw the funicular polygon of Fig. 2d. The line drawn in the $\frac{M_M}{I}$ force polygon (Fig. 2e) parallel to the Y line of Fig. 2d will cut off the R_M reaction of the conjugate beam.

The fixed end moment M_F at the large end will be the quotient of the two conjugate beam reactions.

$$M_F = M_M = -\frac{R_p}{R_M}$$

3. Engr., McConathy, Hoffman & Assocs., New York, N. Y.

Division	I	M_P	$\frac{M_P}{I}$	$M_M = 1$	$\frac{M_M}{I}$
	$(ft.)^4$	ft. kip	$Kip(ft.)^{-3}$		$(ft.)^{-4}$
1	5.00	3.03	0.61	0.054	0.0108
2	5.92	9.08	1.53	0.162	0.0273
3	6.94	15.13	2.18	0.270	0.0390
4	8.20	21.18	2.58	0.378	0.0460
5	9.30	27.73	2.94	0.486	0.0521
6	10.70	33.28	3.11	0.594	0.0555
7	12.10	39.33	3.25	0.702	0.0580
8	13.90	-10.20	-0.735	0.815	0.0585
9	15.90	-3.40	-0.214	0.950	0.0600

T A B L E 1.

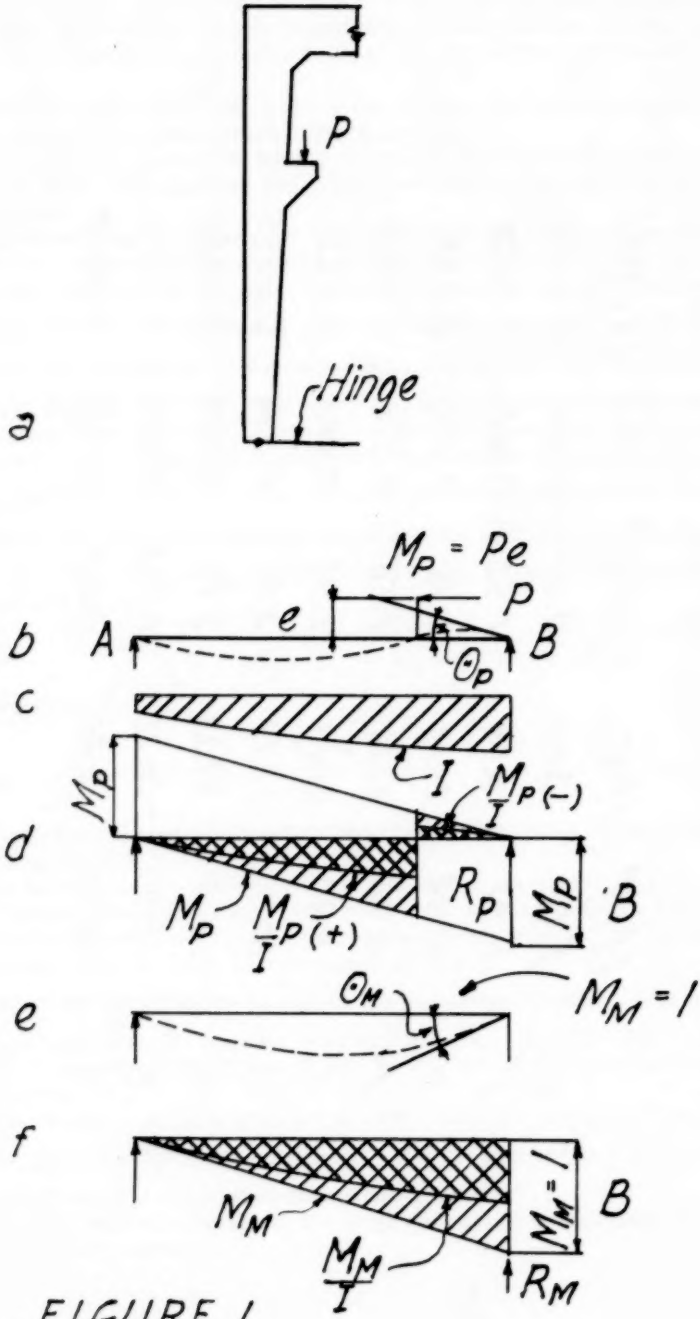
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FIGURE 1.

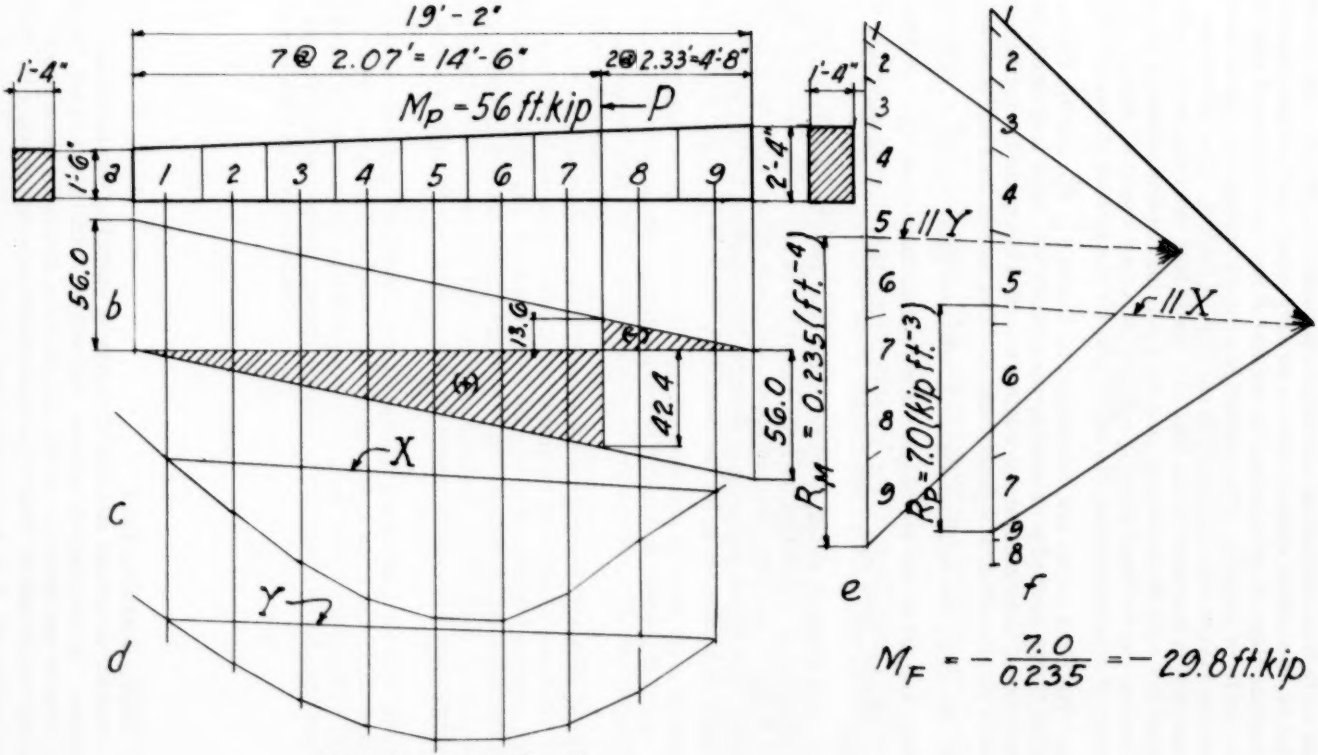

FIGURE 2.

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J. J. POLIVKA,[4] M. ASCE.—The authors mention in the Synopsis that there are other methods of analysis for beams having variable moments of inertia, but more time consuming and subject to numerical errors. One method should not be overlooked which proves to be very simple and permits checking during various steps of calculation, the method of constant centers of elastic rotations (or fixed points[5]). This method has the advantage that a great part of calculation results can be applied to any type of loading. The continuous beam, analyzed in the paper as an example, needs only two fixed points to be determined (because of symmetry), and the resulting bending moments for any type of loading can be found directly with any accuracy required, without using a series of approximations, characteristic for the method of moment distribution. The fixed points are determined by slopes θ^{A}_{A}, θ^{B}_{B}, and θ^{A}_{B}, calculated either by the method presented by the authors, or, more accurately, by the same moment area method, but applied to actual segments having different moments of inertia. There is practically no difference in the results of both methods. So, e. g., the authors' Figure 8 indicates the values $\theta^{B}_{B} = .2663$ and $\theta^{B}_{C} = .1382$, and the writer calculated the corresponding values $\theta^{B}_{B} = .263996$ and $\theta^{B}_{C} = .140636$, their sum being .404632 as compared with the authors' sum .4045.

Every such analysis should be checked for final results. A very simple method consists in calculating the final slopes under any given loading at the supports B and C, for both adjacent spans, and these slopes must be numerically equal. The writer suggests to perform this checking for any of the assumed types of loading, e. g. for the dead load, which is the simplest one.

4. Cons. Engr., Berkeley, Calif.
5. J. J. Polivka, Graphical methods of analyzing statically indeterminate structures. Mimiographed lectures, University of California, 1940, 2nd ed. 1942; Frames and Bents in reinforced concrete, Revue du Béton Armé, Brussels, vol. 9 and 10, 1920; Continuous arches on elastic supports, Der Brueckenbau, vol. 4, 5, and 6, Heidelberg, 1920.

"HIGH STRENGTH STEEL BOLTS IN STRUCTURAL PRACTICE"

by Mace H. Bell
(Proc. Paper 651)

W. HANSEN,[1] A. M. ASCE.—High tensile steel bolts conforming to the A.S.T.M. Specification A325, similar to those described by Mr. Bell, are being used for all the field joints of the Burdekin River Bridge. This Bridge consisting of ten 250 ft. span Pratt trusses is being erected by the Co-Ordinator General's Department.

The high tensile bolts are being used in preference to rivets due to the lack of trained riveters and the advantage of easier scaffolding. Higher initial costs of the bolts has been more than off-set by lower installation costs, particularly in the tropical climate.

As no slip in the joints can be accepted, all faying surfaces are unpainted and conform to the Council's Assembly Specification. The bolts are tightened to the torque required by the Council's Assembly Specification using pneumatic impact Wrenches with the air supply throttled to give "stall" of the wrench at the required torque.

Experiments on the bolts used, with threads of a medium fit to British Standard Specification 84 "Limits of Screw Thread of Whitworth Form," give good agreement with the torque—tension relationship used in the Council's Assembly Specification.

As well as checking the calibration of the wrenches at the stall point, it has been found necessary to check the torque on the bolts in position. About 1 per cent of bolts are checked in this manner with a torque wrench. The fit of the socket on the nut and the rigidity of the member being bolted have been found to affect considerably the tightness obtained on the bolt. A worn socket with a loose fit on the nut fails to give the same tension as a snug fitting socket.

Multiple thickness of plates giving some spring in the joint also give a reduced tension in the bolt below that obtained on a rigid testing device.

Losses in sockets due to cracking and splitting have been large. Could Mr. Bell advise on whether special sockets are in use in U.S.A. and what is the anticipated life of a socket?

1. Engr. in Charge, Burdekin Bridge Erection, Co-Ordinator General's Dept., Queensland, Australia.

"BENDING INTERACTION IN SUSPENSION BRIDGES"

by Haaren A. Miklofsky
(Proc. Paper 652)

N. C. RAAB,[1] M. ASCE.—The initial proportioning of any structure is complicated to a greater degree by the number of elements in the framework and further, in the case of a suspension bridge, by the number of variables to be considered in its analysis.

The author has endeavored to proportion the chord members of the stiffening truss of a suspension bridge by using an interaction diagram. The stresses in the chord members are proportioned from the deflection of the stiffening truss for various assumed loadings. The relationship between the two does offer a solution to the proportioning of the chord area for any particular loading.

The paper would be of much greater value if the relationship between the shear and deflection could be also taken into account to proportion web members.

The author is to be commended for the time and effort devoted to the solution of this complicated problem.

F. P. SHEARWOOD,[2] M. ASCE.—The paper on bending inter action in Suspension Bridges is mainly devoted to devising a formula to visualize the effect of changes in the stiffening trusses of the conventional suspension bridges. However, the ultimate objective must be to design the most effective and economic means of stiffening this special type of bridge; therefore, the writer takes this opportunity of suggesting that the conventional suspended stiffening truss is not the most direct nor best means of resisting the peculiar deflection of suspension bridges.

The paper shows that any sag at one end of the cable must be compensated by a rise at the other end. This action must produce reverse deflections in the two ends of the span. Moreover if the reversals in curvature of the two cables become out of time with each other, torsional deflections in the floor will result.

These distortions in the curvature of the cables can be produced by either uneven vertical loading or by longitudinal forces which tend to swing the suspended structure towards one end. From all reports troubles have not occurred from live load action, but when strong winds have blown diagonally to the plane of the bridges.

A strong wind blowing diagonally to the bridge will produce a considerable longitudinal component force to be carried by the cables to the piers, and unless the cable curve is directly braced, its sag on the windward ends will be increased and that on its leeward end will be straightened out which in turn gives reverse deflections at each end of the span.

1. Project Design Engr., San Franciso Bay Toll Crossings, Berkeley, Calif.
2. Chief of Tech. Staff, United Shipyards Ltd., Dominion Bridge Co., Montreal, Que., Canada.

Unless the independent stiffening member in a suspension bridge is capable by itself of transferring uneven loading or longitudinal forces to their reaction points, some reverse reflections will remain in the floor.

It is these reversals that develop the disturbing undulations and vibrations in many of the existing bridges, while other bridges may have too slim a margin of stability to be exempt from possible trouble, if that fickle unknown force, the wind gusts, should increase its force or alter its numerous directions or frequencies.

It is evident that it is the reverse distortion of the cables curvature at each end of the span that is the cause of trouble. It is therefore desirable to hold the cables to their normal curve by directly trussing them throughout their entire length.

This can be done by making the span into a braced suspension bridge (fig. 1) with the cable as its suspension top chord, the vertical hangers turned into diagonals, having nearly equal inclinations at each panel point and a stiff lower chord to replace the stiffening truss.

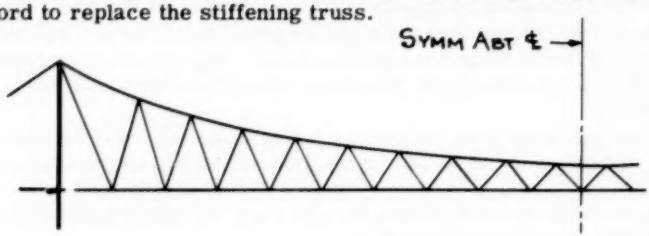

Fig. 1.

Owing to their length and also the connection details, the diagonals must be purely tension members and as the inclinations of each pair meeting at a panel point are nearly equal, the panel load will be nearly equally divided to give equal tension in them.

This braced suspension structure will act as a truss with only direct stresses in all members and a single simple deflection curve until the compressive stress in some diagonal due to an extreme distribution of loading will exceed the tension in it from uniform loading.

This condition will seldom happen and only if the ratio of live to dead load is great and it is not likely to happen and be maintained when very strong winds are blowing. If, and when, it does occur, the curvature of the cable will be slightly deformed until a balance is gained. This distortion can be greatly modified by making the lower chord also capable of resisting the excess shear by cross bending, i. e., using a light stiffening truss proportioned for resting the excess shear as well as its direct stress.

This suggested braced suspended bridge design should prove economical as compared with the conventional suspended stiffening truss design, but its chief merit is that it eliminates any reverse deflections for most conditions of loading especially that due to the longitudinal component of the wind force.

The writer has barely touched on the main part of the paper, but the author's clear illustration of the effect and importance of the cables' distortion prompted the writer to present this discussion.

GEORGE S. VINCENT,[3] M. ASCE.—This paper is a valuable addition to the means available to the designer for the necessary study of a wide range of possible designs for a suspension bridge stiffening truss. The author makes use of the earlier paper by Hardesty and Wessman (his ref. 2), to avoid duplication, in working up certain data, but his interaction diagrams present the relation between truss depth, chord area, stress and deflection in a new and surprisingly simple, clear and comprehensive manner.

The basic idea of the interaction between a beam and a funicular member has been broken down into terms which can be plotted from a few calculations and ingeniously combined by adding reciprocals to obtain three control curves between which intermediate curves can be readily sketched. With these diagrams a clear picture is quickly gained of the effects of a wide range of stiffening truss area and depth, reducing substantially, it is expected, the number of more rigorous analyses to be made.

Of course any such method entails a loss of precision when various parameters must be taken from studies of a range of bridge design characteristics. Moissieff and Ammann pointed this out in discussing the Hardesty and Wessman paper also. However, this does not vitiate the preliminary studies whose main purpose is comparative.

In using this method a large scale is desirable in plotting the interaction diagrams, in which case care is to be taken to observe the geometric scale for the intervals between the curves representing chord area.

Hardesty and Wessman expressed great appreciation for valuable suggestions made by Hardy Cross; Jacob Karol, in discussing their paper acknowledged indebtedness to the same man. Now Miklofsky expresses appreciation of his inspired teaching and guidance. Such evidence of this man's lengthening shadow in the sunset could be given by many other beneficiaries of his inspiration and encouragement.

J. E. ROBINS,[4,5] J. M. ASCE.— An important and easily constructed visual (graphical) tool for design or analysis is presented in the author's paper allowing a designer to compare all possible variations of design from which he can pick the most suitable dimensions for a particular structure. For greatest usefulness such a method must be reduced to its simplest and most direct form.

Two straight lines (m) and (n) can be added with very little effort to the author's interaction diagrams and will greatly facilitate determination of most economical truss dimensions. The condition of most ecnomical truss depth for any given chord area is defined, for the purpose of this discussion, as occuring when the largest value of stress f_t is developed in the chords of the stiffening truss. This results in maximum use of truss chord material for load carrying so the smallest and therefore most economical chord area may be employed. The structural limitation of maximum working stress and any architectural limitations as to maximum deflection and truss depth must, of course, also be met.

3. Highway Bridge Engr., Physical Research Branch, Bureau of Public Roads, Washington, D. C.
4. Ensign, Civ. Engr. Corps, USNR, Naval Air Station, Norfolk, Va.; Formerly Structural Research Engr., David Taylor Model Basin, Navy Dept., Washington, D. C.
5. The opinions or assertions contained herein are the private ones of the writer and are not to be construed as official or reflecting the views of the Navy Department or the Naval Service at large.

Figure 5 (b)[6] shows that a maximum stress f_t will be developed for any given area A when line (a) crosses curve (b), therefore when $I_t = I_c$. From figure 5 it can also be seen that this condition of maximum stress can be plotted, when area A is varied, as a horizontal line (m) on the deflection diagram having a constant relative deflection value of 0.5 and as a corresponding straight line (n) on the stress diagram oriented through the axis with a relative slope equal to 0.5 times the slope of line (a). These two lines (m) and (n) represent an ideal condition, therefore the locus of most economical truss depths for any area A. When a scale, relative or specific, has been established, all lines and curves can be directly transferred, by proportion, from one interaction diagram to another. When the maximum allowable deflection, represented by line (h) on figure 7, is for architectural reasons less than the ideal relative deflection value of 0.5, line (m) must be lowered to the position of line (h). The following rule should be followed: Line (m) should always be superimposed on line (h) when line (h) has a relative deflection value of 0.5 or less, and line (m) should always have a relative deflection value of 0.5 when line (h) has a relative deflection value greater than 0.5. When it is necessary to move line (m) downward from its ideal relative deflection value of 0.5, the corresponding straight line (n) on the stress diagram will pivot clockwise around the axis to a position having a new value of slope, relative to the slope of line (a), numerically equal to the new relative deflection value of line (m) as is easily seen by substituting equation (16) into equation (13); i. e., if line (h) has a relative deflection value of 0.375 this is also the relative deflection value of line (m) and the corresponding line (n) on the stress diagram is a line through the axis with a relative slope of 0.375 times the slope of line (a). Then the most economical design will be the point on line (n) which has the smallest area and further meets the conditions of limiting stress and depth of truss; i. e., the intersection of lines (k) and (n) will be the most economical design unless for architectural reasons a lesser truss depth becomes a limiting factor.

Where the value for area of material in the truss chords as determined by the preceding method does not seem reasonable, a material with a more favorable allowable working stress may be substituted, resulting in a shift of line (k) as described by the author, and the best design can then be redetermined for the revised allowable stress.

It is possible to determine dimensions for the most economical design directly without drawing curves (c) and (d). Place a scale on figure 7 to locate the most economical design stress and truss depth, then solve for area A by substituting the most economical design stress for f_t in the equation on figure 5 (b). In fact it would appear quite simple at this stage to develop equations allowing direct determination of the most economical dimensions without drawing an interaction diagram. That course of development, however, is felt unwise by this writer for the following reasons. Engineers are used to working with graphical representations and can often think in broader terms when employing such a tool. Presenting a stereotyped simplification is dangerous because it may be substituted for creative thinking; that is not the intent of this discussion, but rather it is intended to present a method which may be used in conjunction with the author's development without replacing any part of that development.

Through use, the great advantage of the interaction diagram method becomes exceedingly apparent. As stated in the author's conclusion, an

6. All figures and equations referred to in this discussion are the author's.

interaction diagram can be developed for the arch and in this writer's opinion could also be developed for such interacting structures as a frame and shell ship's hull, a frame and stressed skin aircraft fuselage, a ring reinforced pressure hull, a bent supported structural slab and the architecturally popular frame supported stiffened shell, to name a few possible applications. The advantage of developing an interaction diagram for these structures, as the author has pointed out, is that most suitable dimensions can be determined directly without laborious trial and error. It seems most probable to this writer that as the current literature is assimilated by the structural engineering profession an entire series of new applications for interaction diagrams will develop and come into general use.

SIH, NAN SZE,[7] A. M. ASCE.—It is very interesting to see that the author used the interaction diagram to represent graphically the principle of the deflection theory. The author concludes "Perhaps the most significant contribution of this method for design is that it shows the designer "what to do" to directly determine the most suitable dimensions." It is not a method of trial and error. "However, it does not appear to the writer that by the use of the interaction diagram that the requirements as to the stiffness of the bridge and the truss could be determined directly. Aside from the other considerations pertaining to the design of a bridge, it is the stability or stiffness of a suspension bridge that is the main problem. Once this requirement has been satisfied in the design of a particular bridge, it is a fairly simple matter to carry out the rest of the design, including the proportioning of the stiffening truss with or without the aid of the interaction diagram. Mr. O. H. Ammann in his paper[8] has discussed at length the stability problem of suspension bridges. As in the design of all indeterminate structures, it is a cut and trail procedure. However, by the use of the interaction diagram, the procedure becomes more systematic. The writer suggests that the moment of inertia instead of the chord area shall be used in plotting the diagram, the unit stress in the chord practically varies as the depth of the truss. This can be shown by joining points of the same moment of inertia on the author's diagram 7, which gives a straight line. The same may be done for the deflection curve in diagram 7. The writer also suggests the use of exact equations of deflection theory instead of the approximate methods in making the computations. The equations can be simplified, and by the aid of graphs, the computation can be systematic and not as complicated as it appears. The computation by the approximate method is not direct, and the accuracy of the result can not be confirmed without comparing with results by equations of the deflection theory.

7. Assistant Structural Engineer, Ammann & Whitney, Consulting Engineers, New York, N. Y.
8. Present Status of Design of Suspension Bridge with Respect to Dynamic Wind Action by O. H. Ammann, Journal of the Boston Society of Civil Engrs. July, 1953.

"A STUDY OF THE BEHAVIOR OF LARGE I-SECTION CONNECTIONS"

by J. R. Fuller, T. F. Leahey, and W. H. Munse, Jr.
(Proc. Paper 659)

GLENN B. WOODRUFF,[1] M. ASCE.—Most tests of riveted or bolted connections have been on flat plates. Those reported in this paper are a start towards the more complete investigation recommended by the authors.

Special attention may be given to the following:

1. The lower yield point and ultimate strengths at the "Center-Flange." Mill test specimens are usually cut from the web. In heavy H sections, the discrepancy might be larger than those given in this paper.
2. The specimen test efficiency of less than 75% as against a theoretical efficiency of 83.6%. As the authors state, this efficiency would presumably be less in those cases where lower fabrication standards are used. Again in heavy H sections a larger theoretical efficiency is possible. Pending the results of more complete tests, an efficiency greater than 75% should not be assumed for design.
3. The low stresses at which flange yielding occurred. In the case of steady loading, a stress readjustment would take place. In the case of repeated stresses, such might not be the case. This may be a partial explanation of the failures that have been experienced in hangers of railroad bridges.[2]
4. The more gradual slip of the riveted as compared to the bolted connections (Fig. 4) may indicate a better distribution of the forces between the several connectors.

At present, many investigations under the auspices of the Research Council on Riveted and Bolted Joints are under way. There appears no field where additional research is more required than that suggested by the authors.

1. Cons. Engr., San Francisco, Calif.
2. "Fatigue in Riveted and Bolted Single Lap Joints," A.S.C.E. Separate No. 469.